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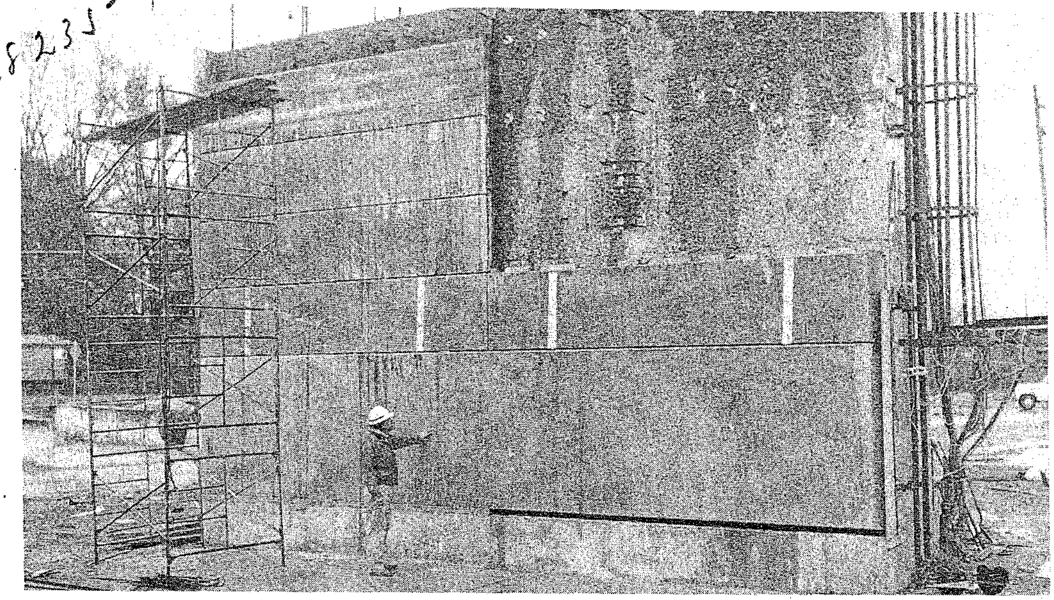
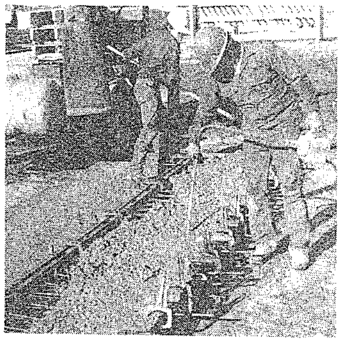
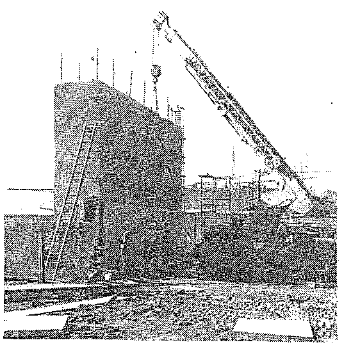
The REMR Bulletin

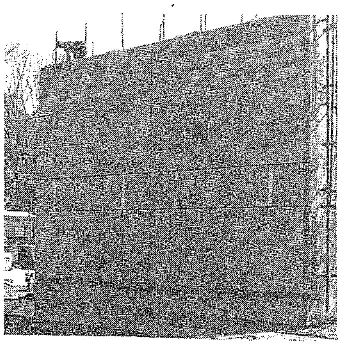
News from the Repair, Evaluation, Maintenance, and Rehabilitation Research Program

VOL 4, NO. 1 **INFORMATION EXCHANGE BULLETIN** **JUL 1987**









Precast Concrete Stay-In-Place Forming System for Lock Wall Rehabilitation

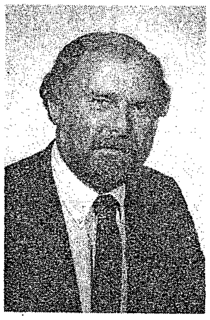
by
James E. McDonald
US Army Engineer Waterways Experiment Station

Approximately half of the 269 navigation lock chambers owned and operated by the Corps of Engineers were built prior to 1940. Consequently, the concrete in these structures does not contain intentionally entrained air and is, therefore, susceptible to deterioration from cycles of freezing and thawing. Since the majority (78 percent) of these older structures are located in the Corps' North Central and Ohio River Divisions, areas of relatively severe climatic exposure, it is not surprising that the concrete in many of these structures exhibits significant deterioration.

The general approach in lock wall rehabilitation has been to remove 1 to 2 ft of concrete from the face of the lock wall and to replace it with air-entrained concrete by using conventional forming and placing procedures. One of the most persistent problems in lock wall rehabilitation with this approach is cracking in the replacement concrete.* These cracks are attributed primarily to restrained contraction of the replacement concrete, the restraint being provided through bond to the stable mass of existing

* McDonald, J. E. 1987. "Rehabilitation of Navigation Lock Walls: Case Histories," REMR Technical Report (in publication), US Army Engineer Waterways Experiment Station, Vicksburg, MS.

REMR RESEARCH PROGRAM



James E. McDonald is a research civil engineer in the Concrete Technology Division, Structures Laboratory, Waterways Experiment Station. He is Problem Area Leader for the Concrete and Steel Structures portion of REMR and is also principal investigator for five REMR work units, including 32273, "Rehabilitation of Navigation Locks." He has been involved with various aspects of concrete research for over 25 years. McDonald received his B.S. and M.S. degrees in civil engineering from Mississippi State University.

concrete. In most cases, such cracking will not cause structural deficiencies; however, the cracks are unsightly and may require additional maintenance to minimize deterioration.

One approach to minimizing the cracking problem developed in the Corps' Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program is to use precast concrete panels as stay-in-place forms. A precast panel rehabilitation system was designed by ABAM Engineers, Inc., Federal Way, WA, in Phase I of a contract with the Corps' Waterways Experiment Station (WES). Phase II was a constructibility demonstration in which eight panels were precast and erected on two simulated lock wall monoliths at WES.

SYSTEM DESIGN

Objectives of the precast concrete stay-in-place form design were to develop a rehabilitation system which provides superior durability, accommodates all of the normal lock hardware and appurtenances, minimizes lock downtime, and can be implemented at a wide variety of project sites. To accomplish these goals, the system was required to satisfy a well-defined set of durability, functional, constructibility, and cost/schedule criteria. Establishing the criteria and baselines for evaluating the performance of the system was an integral part of the design and was used in value engineering analyses of the individual elements of the system.*

* ABAM Engineers, Inc. 1987a. "Design of a Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," REMR Technical Report (in publication), US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Design criteria

Criteria for durability of the rehabilitation system included minimizing the cracking problem in the replacement concrete, providing suitable resistance to cycles of freezing and thawing for the replacement concrete and substrates, assessing creep characteristics of prestressed options, and providing adequate cover or physical protection for reinforcing.

Functionality criteria required that the forming system support the loads imposed by the infill concrete and accommodate all of the normal lock hardware and appurtenances, that the panels accommodate acceptable tolerances on fabrication and erection, and that the panels resist abrasion and impact resulting from normal lock operations.

Constructibility criteria required that the system provide for maximum scheduling flexibility, that it be suitable for use at a wide variety of construction sites and precasting facilities, that panel sizes be favorable to local transportation restrictions, that a maximum of out-of-lock preassembly be accomplished, and that special techniques and equipment usage be minimized.

Criteria for evaluating the cost/schedule of the system were based on a typical 30-ft wide and 40-ft high lock monolith refaced with conventional cast-in-place concrete. Cost and schedule data were developed from repairs performed at the Brandon Road and Lockport locks on the Illinois Waterway and from standard cost estimating and scheduling methods.

Design alternatives

A wide range of alternatives was considered in the design. Alternatives were categorized as materials related, system configuration variables, or panel types and sizes. Material options considered in the design included concrete mixture variables, reinforcing options, and the possible use of surface treatments for the panels. Two primary panel support systems were considered, one consisting of panels supported from the monolith with form ties and a second which used external bracing systems to support the panels during placement of the infill concrete. The precast panels could be oriented either vertically or horizontally with both support systems. Four panel types (hollow core, flat slab, double-tee, and tri-slab) were considered. Also, the use of exceptionally thick or thin panels was considered to ascertain any potential advantage afforded by these options.

Qualitative comparisons

The range of alternatives for achieving the design objectives was evaluated through a process of value engineering. The analysis used a qualitative comparison technique in which the relative merits of the option were evaluated against an arbitrary baseline measure of performance. A value was assigned to the option in each of the four primary criteria categories, depending on whether that option was judged to perform better or worse than the baseline in that category. Based on this analysis, it was concluded that the most advantageous combination of design alternatives was a precast quality concrete ($f'_c \geq 6500$ psi), conventionally reinforced, flat panel, horizontally oriented and tied to the lock wall (Figure 1). Using this system, a detailed quantitative investigation was conducted to determine actual sizes of form panels, tie details, hardware details, etc. These details were then extrapolated to the one-half scale constructibility demonstration which was carried out in Phase II of the project.

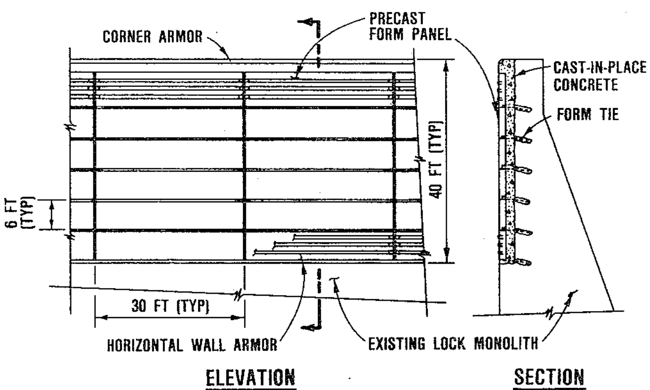


Figure 1. Precast concrete panel system for lock wall rehabilitation

CONSTRUCTIBILITY DEMONSTRATION

The constructibility demonstration involved fabrication and installation of 8 precast concrete panels of varying sizes (Figure 2). The purpose of the demonstration was to evaluate the feasibility of the stay-in-place forming system for lock wall rehabilitation without the risk and investment of undertaking a full-scale lock rehabilitation. ABAM Engineers selected Premier Waterproofing, Denver, CO, as the general contractor for the demonstration. Stresscon Corporation, Colorado Springs, CO, performed the precasting and delivered the panels to Vicksburg, MS, for installation.

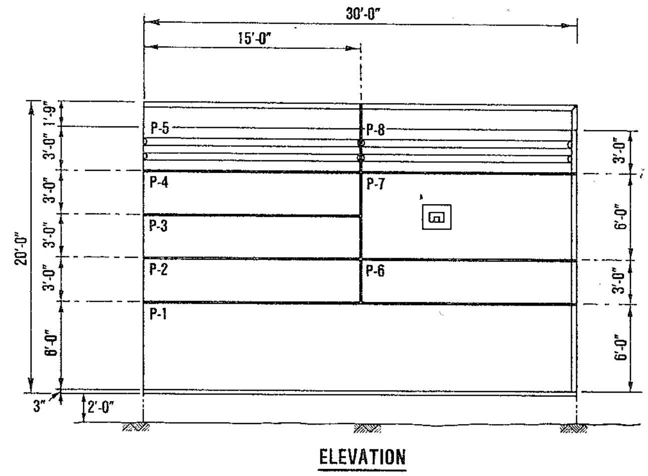


Figure 2. Panels precast and installed in constructibility demonstration

Steel embedment hardware

Approximately 4800 lb of fabricated steel was incorporated into the precast panels. Typical lock hardware included horizontal armor, vertical corner armor, and a one-half scale line hook. Erection hardware included vertical alignment mechanisms, tie weld plates, alignment angles, and shear key plates. All fabricated elements had either headed weld studs or weldable deformed bar anchors for anchorage into the panel. After fabrication, all steel was sandblasted and the exposed surfaces of the lock hardware were coated with a zinc-rich, rust-inhibitive primer.

Panel precasting

The panels were cast on an existing steel form bed which is typically used to cast general flat work pieces such as columns. To ensure a close-tolerance fit between shear keys and to make the forms reusable, 12-gage steel plate was used for the edge forms. The forms were accurately positioned to the panel dimensions and tack welded to the bed surface. The exposed surfaces of all panels except P-5 and P-8 were cast against the steel form bed. These two panels contained horizontal armor which projected approximately 5/8 in. beyond the face of the panel. The precaster placed 5/8-in. thick plywood against the form surface in the space between the armor strips to produce the required projection.

The panels were reinforced with mild steel reinforcing bars and welded wire fabric. Reinforcing bars, No. 4 or 6, were individually placed and tied to form a mat 2 in. from the exterior face.

The 6- by 6-in. wire fabric was placed 3/4 in. from the interior face. Prior to placing concrete, the precaster's project engineer performed a quality control inspection to verify that the formwork was dimensionally accurate and that all required embedments and reinforcing were secured in the required locations. A form ready for concrete placement is shown in Figure 3.

The eight-sack concrete mixture used for the panel precasting was proportioned with high early strength (Type III) cement, 3/4-in. maximum size natural aggregate and a water-cement ratio of 0.38 to 0.40 for a 28-day compressive strength of 6500 psi. Air entrainment and water reducing admixtures were used. Batching and mixing were performed in the precaster's central batch plant. A special truck with a 4-cu yd hopper and auger dispenser was used to transport and discharge the concrete into the forms. The average slump and air content of the freshly mixed concrete were 3.3 in. and 5.2 percent, respectively.

After being placed, the concrete was consolidated with internal vibrators (Figure 4). The concrete was then screeded to the required thickness and finished with magnesium floats. During the floating operation, the C-clamps, which were used to secure embedments, were removed and the resulting depressions filled. As a final step, the concrete surface was raked to obtain a roughened surface on the interior panel face.

The newly cast panels were immediately covered with insulated tarps in preparation for the heat-curing cycle which was initiated approximately 4 hr after completion of finishing. Concrete

Figure 4. Panel precasting

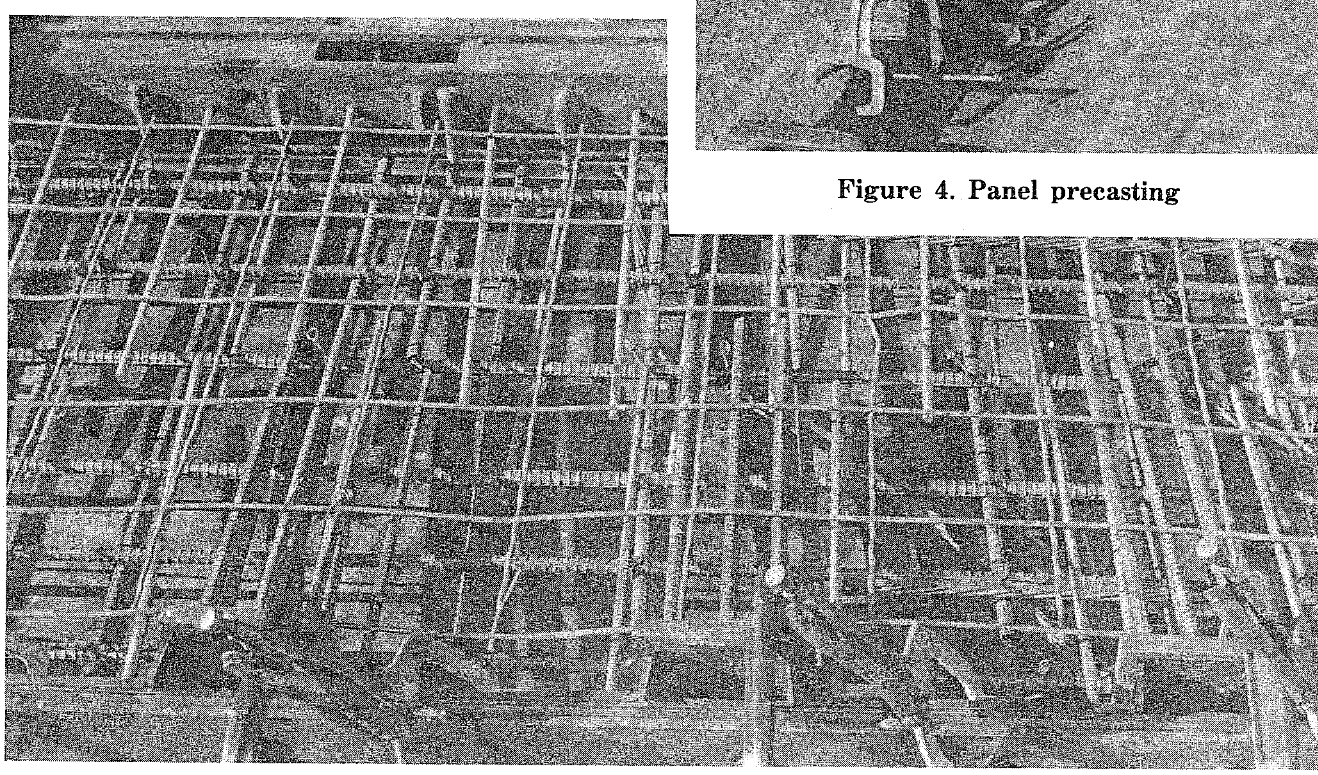


Figure 3. Precast panel form ready for concrete placement

temperatures, monitored by a thermocouple embedded in each panel, reached the 140° to 145° F range after approximately 8 hr of curing. The temperature was maintained within this range until the following morning when the tarps were removed. Ambient temperatures overnight were in the mid- to upper-20° F range.

Panel forms were stripped immediately after removal of the curing tarps, and the panels were lifted off the form bed with a travel lift crane. The compressive strength of the concrete at lift off ranged between 5500 and 6860 psi. The overall average compressive strength was 9040 psi at 28 days age.

With the exception of P-1, all panels were stored in a horizontal position with the exterior face down. Two large timber supports were placed so that positive and negative moments within the panels were equalized. This support minimized deflections and the potential for an adverse permanent set. Because of its length and flexibility, panel P-1 was stored in a vertical position.

All panels were inspected and measured by the general contractor prior to being loaded for shipment to Vicksburg, MS. No cracking was observed and measurements confirmed that the required tolerances were achieved. A flatbed truck was used to transport the panels as their total weight was below the legal load limit. Panels P-1 and P-7 were placed in a steel yoke frame and shipped in a vertical position. The remaining panels were stacked flat, one atop another. The panels were lashed to the truck with slings and come-alongs (Figure 5). Upon their arrival in Vicksburg, the panels were unloaded and inspected for possible damage during shipping. A careful examination failed to disclose any cracking, although some minor localized spalling of Panels P-1 and P-7 occurred at their contact points with the lashing and yoke assembly.

Panel installation

Work associated with installation of the precast panels included surface preparation on the test monoliths, erection and alignment of the panels, welding tie connections, preparation of formwork for cast-in-place (CIP) concrete, and placement and curing of CIP concrete. The general contractor used a basic three-man crew (superintendent, carpenter, and laborer) through the full construction period. This crew was supplemented during the actual erection period with a welder and crane operator.

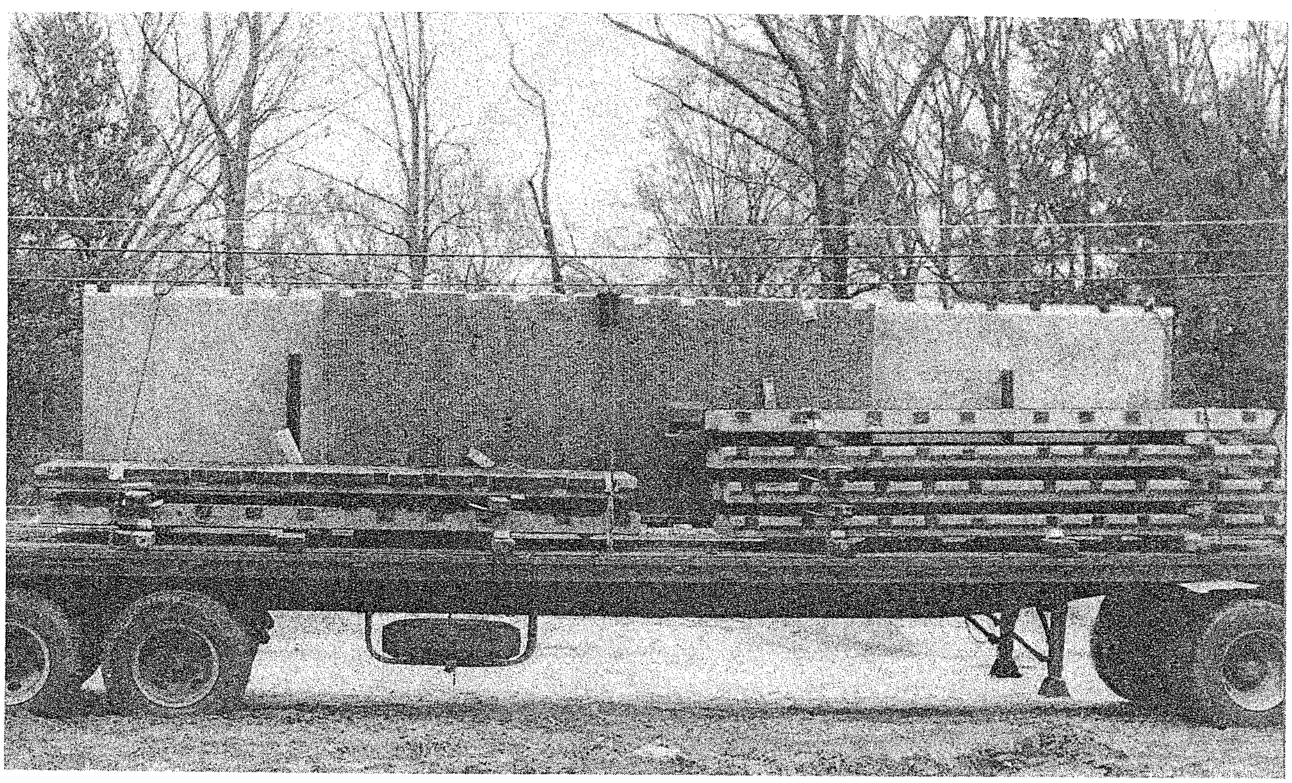


Figure 5. Delivery of panels to installation site

Surface preparation on the test monoliths included installation of 120 form ties and 32 dowels and removal of approximately 16 cu ft of concrete for the line hook embedment.

The high strength of the concrete in the test monoliths and the rough texture of the exposed aggregate surface dictated the drilling procedure for form tie and dowel installation. A small hand chipping gun was used to form a starter pocket, and a pneumatic percussive rotary drill, supported by a rope and pulley arrangement extending from the top of the monolith, was used to finish the hole. Drilling time was approximately 3 min per hole. Concrete removal with a hand-held air powered chipping tool required approximately 30 man-hours.

After the drilling of 1-1/2 in. diameter holes for form ties and dowels was completed, the holes were cleaned with compressed air. A two-component epoxy was then used to grout the 15-in. embedment length for the form ties and dowels which were No. 7 and No. 6 reinforcing bars, respectively. The epoxy was injected with a conventional caulking gun, starting at the back of the hole and working forward to within approximately 3 in. of the wall surface. Insertion and alignment of the reinforcing bars, while the epoxy remained plastic, completed the installation.

During the project planning stages, the contractor anticipated using a 30-ton crane for panel installation; however, a crane of this capacity could not be located in the Vicksburg area. Subsequently, a 15-ton capacity hydraulic crane was located and used to erect the panels, which ranged in weight from 4,000 to 15,500 lb.

Panel P-1, approximately the size and weight of a prototype panel, was installed first. Because of the panel's weight and the limited working radius of the crane, several crane relocations were required to move the panel into final position for erection (Figure 6). Once in position, alignment screws in the bottom of the panel were used to level the panel. Wedges and come-alongs were used as necessary to obtain correct vertical alignment. Following proper alignment, form ties (weldable grade reinforcement) at the top and bottom of the

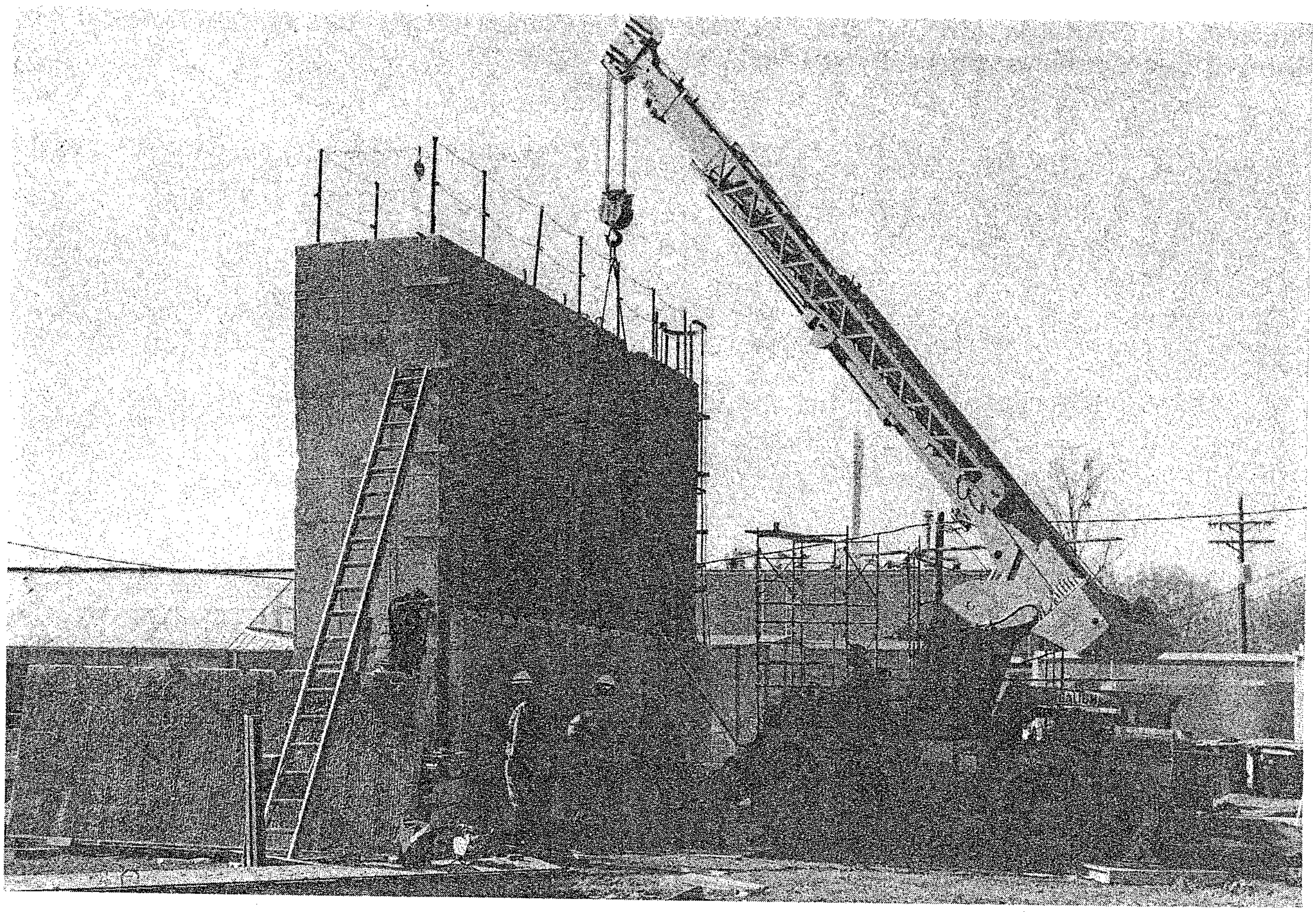


Figure 6. Erection of panel P-1

panel were welded directly to the weld plates embedded in the panel. Initially, heat buildup during the welding process was a problem. Expansion of the embedded steel plates in the top of the panel resulted in numerous fine cracks in the concrete along the base of the shear key. This problem was eliminated by adopting a staggered weld procedure.

Prior to erecting the next panel (P-2), a neoprene seal was bonded to the top bearing surface of panel P-1, shims were installed, and a nonshrink grout was buttered onto the sloping face of the shear key (Figure 7). In addition, the back face of the shear key had to be bushhammered and ground to obtain a proper contact surface for the alignment angle on the succeeding panel. This additional work resulted from an oversight during precasting. Extending the rake finish on the back of the panel to the top of the shear key resulted in an uneven bearing surface which would have prevented a flush alignment between adjacent panels.

Figure 7. Shear key at top of panel P-1 prepared for installation of panel P-2

Once panel P-2 was erected and properly aligned, a finger plate was used to connect the panel and the form ties at the top of the panel. With this connection, the form tie does not project into the plane of the back face of the panel, and thus allows the panels to be installed by lowering them vertically. The steel finger plate spans between the form tie and the weld plate embedded in the panel and is welded to both as shown in Figure 8. A similar detail is recommended for the top connection at the bottom panel instead of the one used on panel P-1.

A similar procedure was used to install the remaining panels with the exception that application of nonshrink grout in the joint was discontinued after the first two joints. It was expected

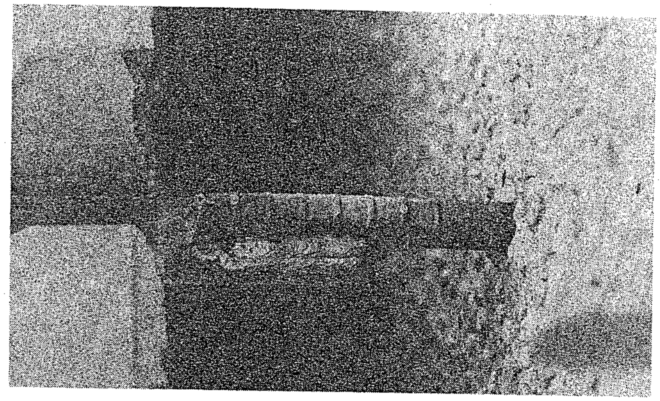


Figure 8. Finger plate connection used to weld form ties

that the grout would flow under the pressure of the upper panel and redistribute to fill any voids along the joint. However, several passes with the trowel were required before the grout stiffened sufficiently to remain on the shear key face. At this point, the grout did not flow under the pressure of the upper panel as expected and actually caused some minor panel misalignment.

The 5-1/2 in. space between the back of the precast panels and the face of the simulated lock monoliths was filled with cast-in-place concrete. On one monolith the infill concrete was placed in single lifts coincident with panel erection. Concrete was deposited from a bucket onto a plywood chute and shoveled into the void as shown in Figure 9. The air-entrained concrete was proportioned with a water-cement ratio of 0.5 for a 28-day compressive strength of 3000 psi. On the second monolith the infill concrete was placed the full 18 ft height of the precast panels in one lift. Concrete was placed with a hopper and an 8 in. diameter elephant trunk. Conventional concrete forming and placing techniques were used to place a concrete cap along the upper 2 ft of each monolith. The completed demonstration is shown in Figure 10.

Inspection of installed panels

A detailed inspection of the precast panels 6 months after installation revealed a number of fine cracks. Because of the extremely narrow width of the cracks, they were not detected until the surface had been thoroughly wet and allowed to dry. Most of the cracks in the precast panel are less than 0.002 in. wide; the widest crack is approximately 0.0025 in. In comparison, a tolerable crack width of 0.006 is suggested for concrete exposed to seawater or seawater spray

Figure 9. Placing infill concrete

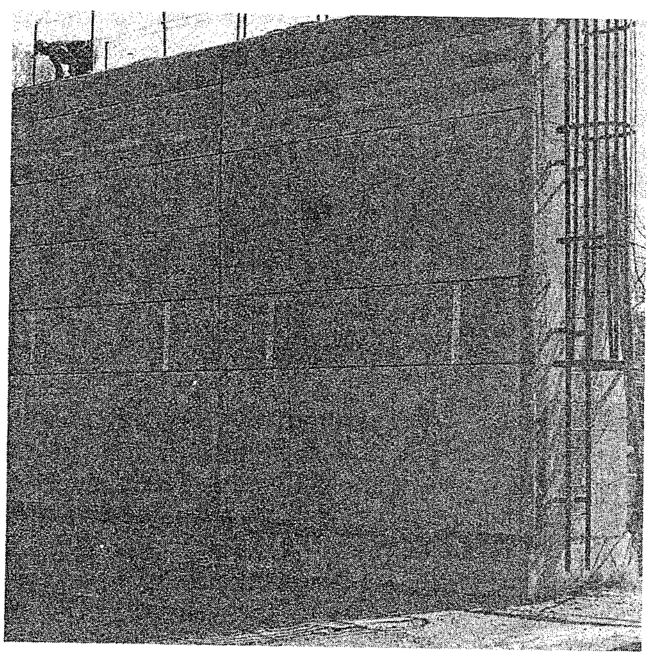


Figure 10. Completed demonstration

under alternate drying and wetting conditions and 0.004 for water-retaining structures.

Cracking appears to have started during precasting, possibly because of curing procedures. With the subsequent shrinking strains and external loads because of handling, shipping, and infill concrete pressures, the cracks have increased in size and become more noticeable. However, they are within industry guidelines relative to the potential for corrosion of the reinforcing steel and do not pose structural concerns.

COST AND SCHEDULE ASSESSMENT

In Phase I the estimated cost of the precast concrete stay-in-place forming system for lock wall rehabilitation was $119/sq ft of lock wall face. In comparison, the cost of conventional concrete forming and placing was $137/sq ft. The actual cost of the demonstration was $151/sq ft, but this cost included only a minimal amount of concrete removal. The small scale of the demonstration installation resulted in higher unit costs to absorb mobilization, general administrative, and engineering costs, some of which will not be proportionately higher for a full-scale repair. In addition, the demonstration installation did not utilize the size of equipment or number of workers that would be optimum for a prototype installation. For example, a crane of the preferred size could not be located and a smaller crane was used; the smaller crane required restricting several of the lifts. Form tie welding was the critical path activity, but only one welder was used, resulting in considerable standby time for the crane and operator.

A construction schedule assessment in Phase I indicated 30 hr would be required to reface a typical lock monolith with the stay-in-place forming rehabilitation system. In comparison, it was estimated that 32 hr would be required for conventional concrete forming and placing. Based on the experience gained during the demonstration installation, it appears that 34 hr is more realistic for the stay-in-place forming system. However, using the precast panel system has the potential for periodic opening of the lock to navigation during rehabilitation, whereas conventional rehabilitation generally requires extended closure and dewatering of the lock.

The demonstration provided not only an evaluation of the overall rehabilitation concept but also a review of specific details incorporated into the design and the efficiency of various work procedures. This review identified a number of items which would enhance future installations.* If incorporated into future prototype lock rehabilitations, these items should enhance both the cost and schedule associated with the stay-in-place forming system.

SUMMARY

Results of this work demonstrate that the precast concrete stay-in-place forming system is a viable method for lock wall rehabilitation. In addition to providing a concrete surface of superior durability with minimal cracking, the estimated construction cost is very competitive with the cost of conventional forming and concrete placement. Also, this system has the potential for significant reductions in the length of time a lock must be closed to traffic for rehabilitation and could eliminate the need for dewatering the lock chamber during wall rehabilitation.

* ABAM Engineers, Inc. 1987b. "Demonstration Installation of a Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," REMR Technical Report (in publication), US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seminar on Automated Instrumentation Data Acquisition for Dam Safety Evaluations

A Corps of Engineers sponsored seminar on all aspects of automated instrumentation data acquisition is planned for September 22-24, 1987, in St. Louis, Missouri. Planned activities include two days of technical presentations, manufacturer displays and an optional field trip to the Clarence Cannon Dam.

The purpose of the seminar is to facilitate technology transfer among Corps field offices and other agencies in all aspects of real time acquisition and management of data used for structural behavior evaluations. Emphasis will be on lessons learned from several "test projects" that used geotechnical and structural instrumentation and are becoming operational in fiscal year 1987. This is an excellent opportunity for any District having experience in automation to present its results.

Prospective session topics are as follows:

- Evaluation of existing instrumentation systems for automation.
- Prioritization of projects and instrumentation to include future expansion.
- System design to include networking and interface with existing automation.
- System implementation to include funding and budget process, procurement procedures, system fabrication, retrofit, integration and maintenance.
- Case histories of "test projects."
- Software and other peripheral applications.
- Research and development efforts.

For more information about the seminar, contact Tony C. Liu (FTS 272-8672 or 202-272-8672) or Arthur Walz (FTS 272-0209 or 202-272-0209).

Seepage Control Seminar Videos Available

"New Remedial Seepage Control Methods for Dams and Soil Foundations" was the topic of a REMR-sponsored seminar held October 21 and 22, 1986, at the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. Video tapes of the seminar are available for loan. The entire set or individual tapes may be requested. Specific topics are listed below.

To request loan of seminar tapes, call WES Library at 601-634-2543 (FTS 542-2543).

The seminar proceedings will be published as a REMR report. For more information on the seminar, call Ed Perry at 601-634-2670 (FTS 542-2670).

Tape	Topic	Speaker	Time Min
1	Introduction	LTC J. R. Stephens, W. F. McCleese, G. B. Mitchell, E. B. Perry WES	30
2	Drains	W. C. Sherman Tulane University	40
3	Upstream Impervious Blankets	W. R. Morrison Bureau of Reclamation	44
4	Use of Hydrofraise to Construct Concrete Cutoff Walls	J. R. Parkison Soletanche	61
5, 6	Plastic Concrete Cutoff Walls	G. J. Tamaro Mueser Rutledge Consulting Engineers	72
7	Jet-Grouted Cutoff Walls	G. Guatteri Novatecna	60
8	Dynamic Grouting by High Speed Liquid Jet	G. Guatteri Novatecna	22
9	Reinforced Downstream Berms	J. M. Duncan Virginia Polytechnic Institute	58
10, 11	Ground Freezing as a Construction Expediency for Cutoff Walls	J. A. Shuster Geofreeze	75
12	Chemical and Micro-fine Grouting	Professor Ruben Karol Rutgers, New Brunswick, New Jersey	58
13	Panel Discussion	J. L. Kauschinger Tufts University	60

Video Report on Remedial Waterstop Installation Now Available

REMR Video Report CS-1, Remedial Waterstop Installation at Pine Flat Dam, is now available to be checked out from the Waterways Experiment Station (WES) Information Technology Laboratory.

This 13-minute video report summarizes a new method of remedial waterstop installation which was used to repair three leaking joints at Pine Flat Dam, California, in 1985. Gelco Grouting Service of Salem, Oregon, performed the repair using a patented system in which they cleaned the drill holes to remove grout from a previous unsuccessful repair, inserted a polyurethane liner which was bonded to the surface of each 6-1/2-in.-diameter hole, and filled each liner with elastic grout to form the core of the waterstop. One year after completion of repairs, the remedial waterstops were still functioning effectively. A detailed description of this repair and several others is contained in Technical Report REMR-CS-4, "Repair of Waterstop Failures: Case Histories."

To borrow a copy of REMR Video Report CS-1, contact the Library of the Information Technology Laboratory, WESIM-TL-R (phone 601-634-4120 or FTS 542-4120).

News in Brief

CPT James G. May has been appointed the Deputy Program Manager for the REMR Research Program at the Waterways Experiment Station, Vicksburg, Mississippi. He has served with the 588th, 548th and 307th Engineer Battalions and the Far East District. He holds a B.S. degree and an M.S. degree in Construction Management from Northeast Louisiana University and Texas A&M University, respectively.

Victor M. Agostinelli has been named to the REMR Field Review Group replacing William R. Hill. He is Chief of the Structures Section for the Engineering Division, Lower Mississippi Valley Division (LMVD). Vic has worked for LMVD for 13 years in the civil works design area. He has a B.S. and M.S. in Civil Engineering from Mississippi State University and Purdue University, respectively.

NEED CORPS STRUCTURES TO DEMONSTRATE WATER JET SYSTEMS FOR REMOVAL OF DETERIORATED CONCRETE: One of the systems to be demonstrated is designed to remove concrete from inclined and vertical surfaces. Other systems are designed to remove concrete from horizontal surfaces only. Demonstrations are scheduled for FY88, preferably between 1 November 1987 and 30 April 1988. For more information, write: Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: WESSC-CE, P. O. Box 631, Vicksburg, MS 39180-0631 or call Roy Campbell at (601) 634-2814 or FTS 542-2814.

A revised list of key personnel for the REMR Research Program is included as an insert to this copy of *The REMR Bulletin*. Save it for a handy reference.

REMR Research Program

KEY PERSONNEL

	Office	Office Symbol	Commercial No.	FTS No.
DRD Coordinator, HQUSACE				
Jesse A. Pfeiffer, Jr.	Civil Works Programs	DAEN-RDC	202-272-0257	272-0257
Overview Committee, HQUSACE				
James E. Crews (Chairman)	Operations Branch	DAEN-CWO-M	202-272-0242	272-0242
Tony C. Liu	Structures Branch	DAEN-ECE-D	202-272-8672	272-8672
Bruce L. McCartney	Hydraulic Design Branch	DAEN-ECE-H	202-272-8502	272-8502
Program Management				
William F. McCleese (Program Manager)	Structures Laboratory, WES	WESSC-A	601-634-2512	542-2512
Cpt. Greg May (Deputy Program Manager)	Structures Laboratory, WES	WESSC-A	601-634-3243	542-3243
Vacant (Technology Transfer Specialist)	Structures Laboratory, WES	WESSC-A	601-634-2587	542-2587
Problem Area Leaders				
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G. Britt Mitchell (Geotechnical—Soils)	Geotechnical Laboratory, WES	WESGE-E	601-634-2640	542-2640
Jerry S. Huie (Geotechnical—Rock)	Geotechnical Laboratory, WES	WESGR-M	601-634-2613	542-2613
Glenn A. Pickering (Hydraulics)	Hydraulics Laboratory, WES	WESHS-L	601-634-3344	542-3344
D. D. Davidson (Coastal)	Coastal Engineering Research Center, WES	WESCW-R	601-634-2722	542-2722
Ashok Kumar (Electrical and Mechanical)	Construction Engineering Research Laboratory	CERL-EM	217-373-7235	958-7235
Jerome L. Mahloch (Environmental Impacts)	Environmental Laboratory, WES	WESEP-W	601-634-3635	542-3635
Anthony M. Kao (Operations Management)	Construction Engineering Research Laboratory	CERL-EM	217-373-7238	958-7238
Field Review Group				
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James C. Wong	New England Division	NEDOD-P	617-647-8411	839-7411
Stanley R. Jacek	North Central Division	NCECO-O	313-226-6797	226-6797
John J. Sirak, Jr.	Ohio River Division	ORDCO-M	513-684-3418	684-3418
Carl F. Kress	South Pacific Division	SPDCO-O	415-556-8549	556-8549
Neal H. Godwin, Jr.	Southwest Division	SWDCO-O	214-767-2429	729-2429
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12/86

REMR Reports Published to Date

Number	*Date*	*Title*	*AD Number*
Unnumbered	Feb 1983	REMR Research Program Development Report, by J. M. Scanlon, Jr., J. E. McDonald, C. L. McAnear, E. D. Hart, R. W. Whalin, G. R. Williamson, and J. L. Mahloch	AD A125 998
Unnumbered	Sep 1985	The REMR Notebook	
Unnumbered	Jan 1987	Proceedings of REMR Workshop on Assessment of the Stability of Concrete Structures on Rock, 10-12 September 1985, compiled by W. F. McCleese	
TR REMR-CS-1	Sep 1984	Engineering Condition Survey of Concrete in Service, by R. L. Stowe and H. T. Thornton, Jr.	AD A148 893
TR REMR-CS-2	Apr 1985	The Condition of Corps of Engineers Civil Works Concrete Structures, by J. E. McDonald and R. L. Campbell, Jr.	AD A157 992
TR REMR-CS-3	Jul 1986	Latex Admixtures for Portland Cement Concrete and Mortar, by D. L. Bean and T. B. Husbands	AD A171 352
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TR REMR-CS-5		Instrumentation Automation for Concrete Structures	
	Dec 1986	Report 1 Instrumentation Automation Techniques, by John Lindsey, David Edwards, Aubrey Keeter, Tom Payne, and Roger Malloy	AD A178 139
TR REMR-EI-1	Nov 1986	Applicability of Environmental Laws to REMR Activities, by J. E. Henderson and L. D. Peyman	AD A177 322
TR REMR-EI-2	Nov 1986	Bibliography of Environmental Research Related to REMR, by N. R. Nunnally	AD A177 069
TR REMR-GT-1	Sep 1984	Mathematical Analyses of Landside Seepage Berms, by R. A. Barron	AD A150 014
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TR REMR-GT-3	Aug 1986	Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels, by C. P. Cameron, K. D. Cato, C. C. McAneny, and J. H. May (Report 1 of a Series)	AD A173 163
TR REMR-HY-1	Jul 1986	Annotated Bibliography for Navigation Training Structures, Compiled by W. E. Pankow and R. F. Athow, Jr.	AD A173 303
TR REMR-OM-1	May 1986	Evaluation of Existing Condition Rating Procedures for Civil Works Structures and Facilities, by Enno Koehn and A. M. Kao	AD A170 391
TR REMR-CO-1		Stability of Rubble-Mound Breakwater and Jetty Toes	
	Dec 1986	Report 1 Survey of Field Experience, by D. G. Markle	

Reports listed having AD numbers can be purchased from the National Technical Information Service, US Department of Commerce, 5285 Port Royal Road, Springfield, VA 22161; telephone (703) 487-4600. Costs of hard copies or microfiche copies of these reports are available from NTIS on request.

REMRNET - Now Online

The REMR research program recently established REMRNET as its latest technology transfer aid. REMRNET is an electronic bulletin board information system which is a part of the Corps' Research and Development Bulletin Board. It is available to anyone with a computer or terminal which is capable of communicating with the Corps' OnTyme Electronic Mail System through the Tymnet Communications Network. REMRNET allows instant access to many aspects of the REMR research program. It can also be used to solicit information for a problem or to share expertise with someone else to help solve his or her problem.

REMRNET is menu driven and requires no computer expertise — i.e. very user friendly. The main menu provides access to REMRNET's 4 major areas:

Available REMR Products

Scheduled Field Tests and Demonstrations

Upcoming REMR Program Events

Interactive Problem/Recommendation Input

The first area, Available REMR Products, lists the available REMR reports, videos, bulletins, etc. by problem area: Concrete & Steel, Geotechnical, Hydraulics, etc. Once the problem area is selected the products for that area are displayed. Information provided includes the product number, title and author. Ordering information is also available.

The second major area lists upcoming REMR field tests and demonstrations according to the Corps' Division in which they will take place. Information is categorized by division. The field tests for that area are displayed along with the date, title, work unit number and location.

The third major area, Upcoming REMR Program Events, includes scheduling information on upcoming training courses, workshops and meetings dealing with REMR technology. The date, name and location are provided.

The fourth major area is an interactive problem and recommendation bulletin board. In this area existing problems and recommendations can be reviewed, recommendations for an existing problem can be entered or new problems can be submitted for others to see and make recommendations. New problems and recommendations can be entered directly to the REMRNET workspace or by uploading an ASCII file created on a word processing package such as Word Star, Framework or Symphony. More information about uploading files can be obtained from a local OnTyme Coordinator.

REMRNET was designed by Ed O'Neil from the Concrete Technology Division, Structures Laboratory, Waterways Experiment Station. Actual development was performed by Gregg Hoge, a computer systems analyst with the Information Systems Branch at CRREL, because of his previous experience with the design of similar systems. The format for REMRNET is so clean and simple that it has been adopted by McDonnell Douglas as its standard for government bulletin boards.

REMRNET is accessed by entering the OnTyme system and using the following ID and Key:

ID? CORPS.DAENRDBB

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Then the Research and Development Bulletin Board Information System will present a menu of applications. Choosing item number 5 on the main menu will access the REMR Network System. It's that easy. Information about the OnTyme system can be obtained from a local OnTyme coordinator. Problems or recommendations concerning REMRNET should be addressed to the REMRNET monitor (CPT Greg May) at FTS 542-3243, commercial (601) 634-3243 or by leaving a message for OnTyme I.D. CORPS.WESSCA.

Remedial Measures to Control Excessive Leakage at Richard B. Russell Dam

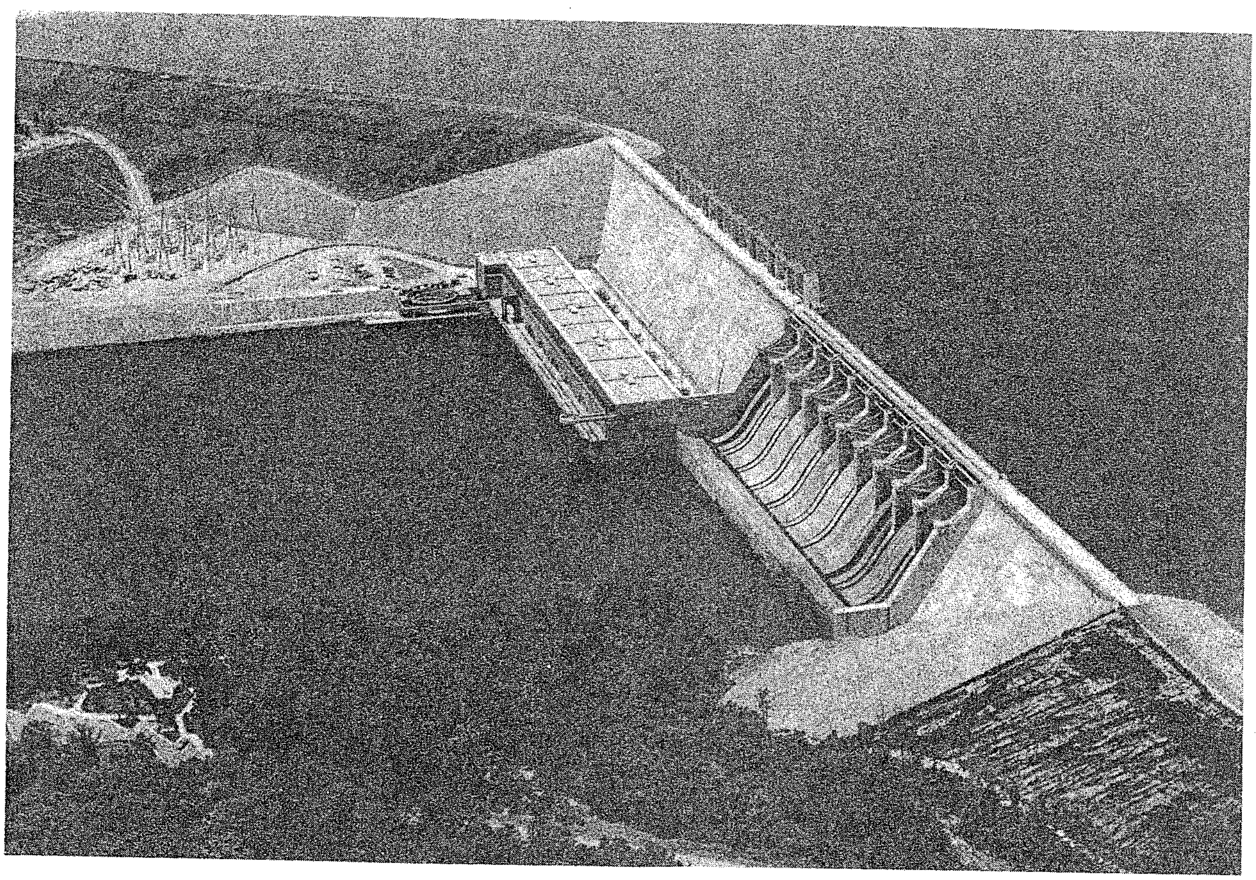
by
Gary Close and John Hager
US Army Engineer District, Savannah

Richard B. Russell Dam is a concrete gravity structure with earth embankments located on the Savannah River between Georgia and South Carolina. The 32 concrete monoliths of the dam include a 600-foot power intake section, a 590-foot gated spillway section, and nonoverflow sections of 288 and 336 feet, resulting in a length of 1,884 feet. The top of the dam is at elevation 495, approximately 200 feet above the riverbed. Under normal operation, the pool ranges between elevation 470 and 480.

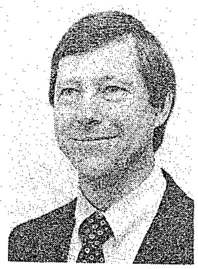
In 1982, with the pool still being held low for the construction-diversion stage, minor leakage was observed coming from the interior face and contraction joint drains into the inspection gallery. This leakage occurred with only 35 feet of head against the dam when the reservoir was at levels as low as elevation 330. When higher-than-normal rain runoff from upstream dams caused the Russell pool, while still in the diversion stage, to reach elevation 381, leakage became significant. Since the operating pool level is 94 feet higher than elevation 381, a plan to quantify and treat the leakage problem prior to filling the lake became imperative.

The final plan included the following steps:

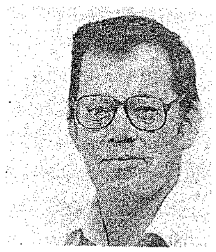
- Raise the pool to elevation 400 to quantify the rate of leakage.
- Test to determine locations and causes of and corrections for leakage.
- Make initial test repairs.



Richard B. Russell Dam



Gary Close is a Structural Engineer in Savannah District's Structural Section. He received his B.S. degree in civil engineering from the University of South Carolina. At Savannah District, he is responsible for civil and military design. He has been with the Corps of Engineers since 1973.



John Hager is a Structural Engineer in the Savannah District's Structural Section. He received his B.S. degree in civil engineering from the University of Minnesota. He was the senior design engineer on the Richard B. Russell Concrete Dam and Powerhouse. He has worked with the Corps of Engineers since 1961.

- Raise the pool to elevation 400 to determine the effectiveness of initial test repairs.
- Execute additional testing as required.
- Make preventive leakage repairs.

TESTING PROGRAM

When the pool was raised to elevation 400, monitoring revealed leakage of approximately 600 gal/min through the concrete dam into the galleries. Based on these figures, projections for full pool elevation indicated a leakage rate in excess of 3,000 gal/min, a rate considered unacceptable.

A number of different types of tests were conducted to determine the location and cause of the leakage and to develop a repair scheme. Among these were smoke tests which were run in the galleries. Various colors of smoke were injected with compressed air into preselected face drains. At the same time, an Inframetrics infrascope with a temperature sensitivity of 1°C and a telephoto lens was installed in a boat to video record and "colorize" the results. The smoke tests had limited success.

Compressed air tests were conducted by sealing the bottom of the face drains and the top and bottom of the joint drains and then piping compressed air into the drains. The amount of air flow needed to maintain a constant pressure within the drain was measured.

Water tests consisted of sealing the bottom of the drains with packers and then filling the drains with green water in incremental steps. At each step the flow required to hold water at that given level was measured. A video camera team suspended over the side of the dam recorded evidence of leakage.

The pulse-echo method was also used to test the entire concrete dam. Using a 5-foot vertical and horizontal grid system, testers struck the concrete with a hammer and recorded the vibrations as they traveled through the concrete.

Finally, ultrasonic velocity investigation was used to delineate areas of poor-quality concrete. An ultrasonic energy transmitter inside the gallery read signals from a matching receiver placed at various elevations on the upstream face of the dam. Interruptions, scatter, or discontinuity of the signal indicated cracks, honeycomb voids, etc.

TEST CONCLUSIONS

Most of the face drains having significant leakage were adjacent to monolith vertical contraction joints and were generally in the latter placed monolith of any given two monoliths. This finding indicated some difficulty in placement or consolidation or both of concrete in the second monolith. This and other correlative data showed that water was entering the vertical contraction joints and traveling horizontally along lift lines or honeycomb voids to the face drains. The flow traveled around the contraction joint copper waterstops and into the contraction joint drains.

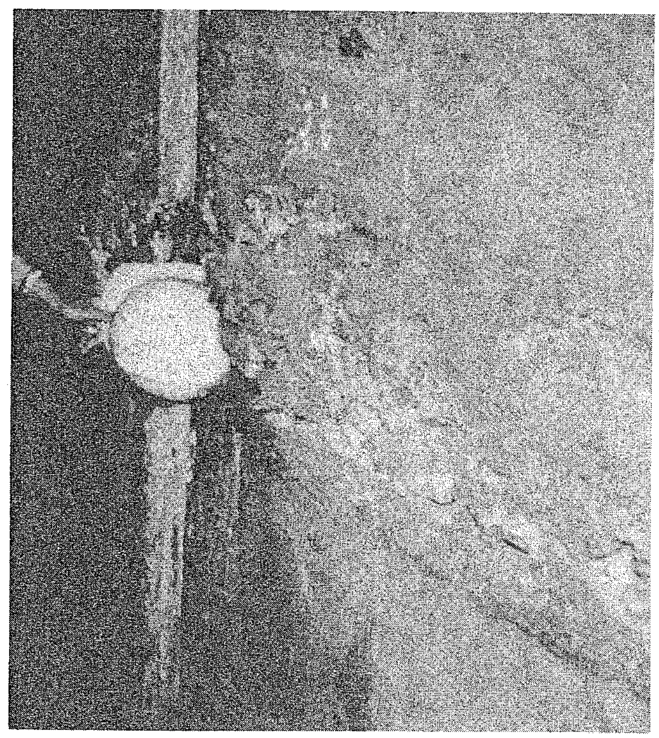


Figure 1. Poorly consolidated concrete adjacent to face drain, July 1980, Richard B. Russell Dam

Most of the leaking drains were located in the power intake section. Most of the leakage was entering the penstock intake opening areas at approximately elevation 380. A minor amount of leakage originated from areas of poorly consolidated concrete on the upstream face of the dam.

An investigation to determine problems experienced during concrete production and placement and their causes revealed that the contractor had had difficulty maintaining his concrete batch plant throughout the life of the project. Also, sand moisture and aggregate gradation problems occurred frequently. However, the dam was thoroughly inspected externally and internally with pulse-echo and ultrasonic velocity instruments and found to be structurally sound and capable of performing as designed.

INITIAL TEST REPAIRS

Initial test repairs consisted of:

- Installing elastomeric grout seals at very low elevations on all vertical contraction joints between monoliths 3 and 30.
- Installing similar seals on the upper ends of vertical joints in the spillway section.
- Placing full-length elastomeric seals in the five vertical joints suspected of accounting for the greatest leakage inflows.

An elastomeric grout was used because of expansion and contraction of the concrete monoliths.

After the test repairs on the five joints were completed, the pool was raised to elevation 400 to test the effectiveness of the repair. Inspection revealed that leakage into four of the grouted joints was either eliminated or drastically reduced.

PHASE I REPAIRS

Based on the success of the initial test repairs, Phase I procedures (repairs prior to reservoir filling) were begun. All vertical contraction joints, except the two end joints on each abutment, were fully sealed as a precautionary measure. Because the pool had been raised only to elevation 400 and the water and initial smoke tests had indicated leakage potential from the upstream dam face and through vertical joints above elevation 400, the sealing was installed upward to elevation 490.

To ensure the effectiveness of the elastomeric seals installed during initial test repairs, a 4-inch-diameter hole was cored into the bottom elastomeric seal at the contraction joint from the dam face to 4 inches beyond the contraction joint copper waterstop. This cored hole was filled with a polysulfide elastomeric grout.

A round elastomeric grout tube, filled with a flexible epoxy sealant, was placed in the vertical contraction "V" joint. The "V" joint was then filled with polysulfide sealant until flush with the dam face. A 20-gage, 16-inch-wide stainless steel sheet metal waterstop was installed across the contraction joint. The waterstop extended from the lower polysulfide grout core upward to elevation 490. Elastomeric grout was used to seal the waterstop to the contraction joint. Contraction joints in the spillway section were sealed into the internal copper waterstops on the spillway crest as well as the lower joint seals.

Leakage from honeycomb areas on the dam face was controlled by sealing the surface. The leakage areas were less than 1 square foot. The affected areas were chiseled out approximately 1 inch deep, coated with epoxy, and, while still wet, filled with nonshrink epoxy grout. A mixture of one part sand to one part epoxy was used to patch porous and honeycomb areas.

PHASE II TESTING AND REPAIRS

Phase II testing and repair efforts (after reservoir filling) varied with the types of leakage. Water dye tests and compressed air tests were used to pinpoint external leakage into the concrete dam. The two methods were used with varying degrees of success.

To verify whether leakage was occurring at the contraction joints beneath the bottom elevation of the stainless steel seals or around the stainless steel seals at the lift lines, a continuous-sheet rubber seal, approximately 2 feet wide, was placed over the lower unsealed contraction joints and any leaking stainless steel seals and lift lines. Contraction joints 10/11 and 15/16, with leakage rates of 78 and 100 gal/min, were selected for testing.

Joint 15/16 was not suited for the rubber seal because the upstream faces of the two monoliths were not flush, so joint 10/11 was used. However, the bonding adhesive did not hold. Next, external expansive elastomeric grouting from the reservoir was tested on joint 15/16. Half of the block 15 face drains had been sealed off with packers, reducing flow to 10 gal/min; the water temperature was approximately 40°F. The joint grouting operation did not work because the inflow was too fast and the water too cold for the CG 5600, a hydrophylic polymer manufactured by 3M Company, to set up and bond with the concrete. The entire operation proved to be very difficult and was only partially effective.

Fiberglass strands covered with Splash Zone, an epoxy resin, were used by divers to seal the contraction and lift line joints and the lower unsealed contraction joints in the power intake and the spillway. Fiberglass-epoxy patches were used to seal honeycomb leakage areas. The epoxy resin displayed superior qualities for adhering to concrete under water. Leakage was reduced 142 gal/min in these joints.

Interior grouting was considered the most effective way to control the leakage at contraction joint 4/5 and face drain 15F. Face drain 15F was plugged with grout injected through a packer. Repair at joint 4/5 consisted of drilling holes beside the drain and angling them into the suspect leakage path. Expansive elastomeric grout injected into the areas was effective in stopping contraction joint leakage and in reducing leakage into an adjacent face drain. This procedure was successful because the high leakage occurred midway up the inspection gallery wall.

Face drain 24C, which leaked at approximately 100 gal/min in October 1983, was sealed incidentally when an emergency bulkhead was placed at the adjoining sluiceway to perform a dye test. The bulkhead in the sluiceway stopped the leakage at 24C. Investigation uncovered four or five honeycomb areas in the ceiling around the access. These areas were grouted with CG 5600.

Some leakage occurred in the penstock perimeter in monolith 14 at elevation 400. This leakage was controlled by grouting the space between the penstock liner and interior near the area where the liner exits the dam. Grout was injected into a "ring" of holes drilled in the liner 6 feet from the downstream face of the dam. A second grout ring was installed 5 feet upstream from the first. As a precautionary measure, all penstock perimeters were grouted in a similar manner.

Silva-Cel Wood Fibers, cinders, and bentonite (which had been used at other dams) had no noticeable effect on stopping the leakage. Grouting from the gallery interior was effective, but keeping the grout out of the drains was difficult. The fiberglass-epoxy resin strands and patches produced very good results. Phase II repairs reduced leakage from approximately 950 gal/min to 350 gal/min. Furthermore, it is believed the repairs kept the leakage from increasing substantially as the pool was raised further.

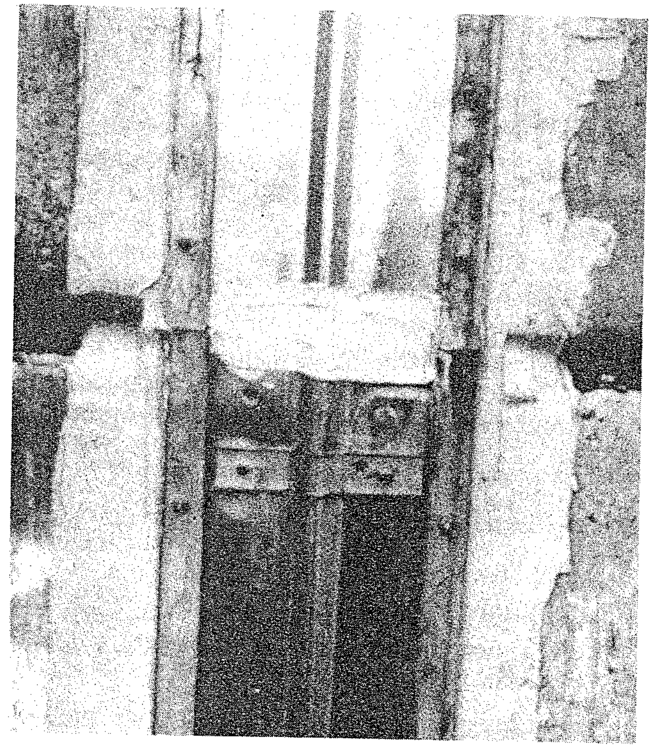


Figure 2. Upstream face of monolith joint with completed external waterstop, Richard B. Russell Dam

INCREASE IN SUMP PUMP CAPACITY

The initial sump pump capacity for each of two sumps was 500 gal/min. The sump pump capacity was increased by replacing the existing vertical turbine pump and motor with one of a larger size. A total capacity of approximately 1,125 gal/min was obtained for each sump in the drainage system.

CONCLUSIONS

The remedial repairs were effective in controlling and reducing the concrete dam leakage. Leakage was to be observed from January 1, 1985, to January 1, 1986, before additional repairs would be considered. Data from other dams indicate that the formation of calcium carbonate will further reduce the leakage.

For additional information on the remedial measures for excessive leakage at the Richard B. Russell Concrete Dam, contact John Hager at FTS 248-5256 or COM (912) 944-5256 or Gary Close at FTS 248-5587.

COVER PHOTOS

Panel precasting
Erection of panel P-1
Completed demonstration

SEP 14 1987

The REMR Bulletin

The REMR Bulletin is published in accordance with AR 310-2 as one of the information exchange functions of the Corps of Engineers. It is primarily intended to be a forum whereby information on repair, evaluation, maintenance, and rehabilitation work done or managed by Corps field offices can be rapidly and widely disseminated to other Corps offices, other US Government agencies, and the engineering community in general. Contributions of articles, news, reviews, notices, and other pertinent types of information are solicited from all sources and will be considered for publication so long as they are relevant to REMR activities. Special consideration will be given to reports of Corps field experience in repair and maintenance of civil works projects. In considering the application of technology described herein, the reader should note that the purpose of *The REMR Bulletin* is information exchange and not the promulgation of Corps policy; thus, guidance on recommended practice in any given area should be sought through appropriate channels or in other documents. The contents of this bulletin are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. *The REMR Bulletin* will be issued on an irregular basis as dictated by the quantity and importance of information available for dissemination. Communications are welcomed and should be made by writing the Commander and Director, US Army Engineer Waterways Experiment Station, ATTN: T. D. Ables (WESSC-A), PO Box 631, Vicksburg, MS 39180-0631, or calling 601-634-2587 (FTS 542-2587).

DWAYNE G. LEE
Colonel, Corps of Engineers
Commander and Director

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